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The Trusted Integrator for Sustainable Solutions

September 23, 2014

Scott Hansen, SR-6J
Remedial Project Manager
United States Environmental Protection Agency
Region 5 Offices
Ralph Metcalfe Federal Building
77 West Jackson Blvd.
Chicago, IL 60604-3590

Mr. Hansen:

WESTON is pleased to present herein our Technical Submittal which addresses the technical feasibility of the dry excavation construction approach currently being considered by EPA and the Wisconsin Department of Natural Resources (WDNR) to remediate contaminated offshore bay bottom sediments within Chequamegon Bay of Lake Superior near the city of Ashland, Wisconsin. This Technical Submittal includes an Executive Summary, a Technical Report, two Technical Discussions prepared by Dr. William L. Deutsch Jr., Ph.D., P.E., and nine (9) Technical Exhibits. The Technical Submittal documents in detail all aspects of the geotechnical analyses completed by WESTON that are relevant to our determination of the technical feasibility of the dry excavation remediation approach. It is noteworthy that the Technical Discussions were prepared to address, in significant technical detail, the two most serious potential geotechnical instability scenarios of "excavation bottom basal heave" and "excavation bottom uplift" relevant to a dry excavation remedial action at the Ashland project site. Both of these technical concerns are exacerbated by the presence of artesian pore water pressure within the predominantly granular Copper Falls geologic formation which underlies the project site. It is noted that the initial submitted drafts of these Technical Discussions were subsequently modified to the final versions presented herein consistent with a review and critique of these documents by WDNR, EPA, and Dr. Joseph Schulenberg, Ph.D., P.E., a Regional Technical Specialist with the Chicago District of the U.S. Army Corps of Engineers (USACE).

Based on our technical work effort and the discussions and analyses results presented in this Technical Submittal, WESTON is pleased to report that we have concluded that removal of the near shore contaminated sediments that exist at and below the mud line of Chequamegon Bay can be safely completed using a dry excavation remedial construction approach which will be protective of both human health and the environment. In this regard, it is important to note that successful and safe completion of a dry excavation remedial action at the project site must be predicated on the assumption that a detailed final engineering design be completed by an experienced, qualified engineering design firm who will develop construction level Design Drawings and Project Specifications consistent with the conclusions and preliminary design recommendations presented herein. In addition, a safe and successful dry excavation remedial construction must also be predicated on the use of qualified, competent, experienced construction contractors to complete the various components of the construction. Finally, it is assumed that necessary field construction quality assurance (CQA) services will also be provided by qualified, experienced Field Engineers or Engineering Inspectors preferably employed by the engineering design firm.

WESTON considers the components of this Technical Submittal to be in Final form. We also note that the Technical Submittal includes the Professional Engineer (P.E.) seals of Dr. Deutsch, Ph.D., P.E., a Technical Consultant to WESTON, and Mr. Adam Brown, P.E., WESTON's Lead Geotechnical Engineer for the project. Their seals are an indication of both their intimate involvement in the preparation of the submittal, and their professional concurrence with the conclusions and preliminary design recommendations presented in these documents.

We sincerely appreciate the opportunity to be of continued service to EPA on this interesting and challenging project. Should you have any questions, comments, or concerns regarding this Technical Submittal, please do not hesitate to contact us.

Sincerely,

Adam Brown

Adam Brown, P.E.
WESTON Senior Geotechnical Engineer

William L. Deutsch Jr.

William L. Deutsch Jr., Ph.D., P.E.
Geotechnical Consulting Engineer
(Subcontractor to Weston Solutions, Inc.)

Omprakash S. Patel

Omprakash Patel
WESTON Project Manager

Enclosures:
1) Technical Submittal

cc: none

Final

ASHLAND LAKEFRONT SUPERFUND SITE TECHNICAL SUBMITTAL

Prepared for:

United States Environmental Protection Agency

Prepared by:

Weston Solutions, Inc.

Dr. William L. Deutsch Jr, Ph.D., P.E.

Geotechnical Consulting Engineer

and

Adam Brown, P.E.

Principal Project Engineer

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2014

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William L. Deutsch Jr.

9/23/14

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EXECUTIVE SUMMARY

EXECUTIVE SUMMARY

The following discussion introduces the reader to the Ashland Lakefront Superfund Site (Site) and discusses why the use of dry dredging[1] technology to remove contaminated offshore bay bottom fill materials and sediments that underlie the free water of Chequamegon Bay can be safely and effectively completed at the Site. Accompanying this Executive Summary is a more detailed Technical Report that further discusses the geotechnical engineering analyses completed by Weston Solutions, Inc. (WESTON®) which support our position regarding the applicability and suitability of dry dredging technology at the project site.

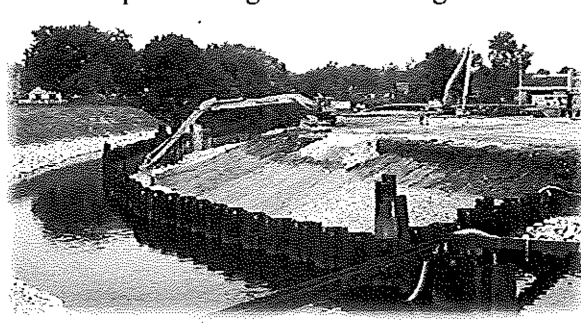
THE ASHLAND LAKEFRONT SUPERFUND SITE AND LOCAL SUBSURFACE ENVIRONMENT

The Ashland Lakefront Superfund Site is located in the town of Ashland, Wisconsin, along the southeastern banks of Chequamegon Bay of Lake Superior. It encompasses a recreation area known as Kreher Park, an upland bluff, a former wastewater treatment facility (now demolished), and an approximately 16 acre offshore area within the bay. All of these areas have experienced extensive contamination typically produced by the byproducts of the manufacture of heating and lighting gas, i.e. hydrocarbons and heavy tars. The focus of this discussion is the offshore portion of the Site.

The local subsurface environment consists of contaminated fill materials from historic lumber operations which are primarily wood waste and contaminated bay bottom soft sediments that directly underlie the free water of the bay. These materials are underlain by the natural very low permeability soils of the Miller Creek Formation (MCF). These soils consist of layers of hard silts and soft to medium stiff clays. The MCF soils overlie the Copper Falls Formation (CFF), a highly permeable soil stratum that contains dense sands and gravels that extend to great depth. As discussed below, the CFF is also a regional artesian aquifer.

WESTON TECHNICAL ANALYSES

The technical issues relevant to an assessment of the technical viability of a dry dredging remedial action are largely geotechnical in nature and therefore require proper assessment by a qualified, experienced geotechnical engineer. A



[1] Dry dredging is also referred to as dry excavation. Both terms refer to removal of subsurface materials (e.g. contaminated sediments) which exist beneath a free water body "in the dry" via installation of a retaining structure (i.e. a cantilevered steel sheet piling wall) to separate the dry dredging footprint from the free water body, followed by dewatering of the footprint before conventionally excavating the targeted materials.

primary concern of the project geotechnical engineer is the health and safety of construction personnel during completion of the dry excavation field work. In this regard, WESTON has identified five potential instability scenarios that are geotechnical in nature which could develop in the field during the dry excavation field work. These instability scenarios therefore require proper and thorough evaluation and quantitative analysis before the technical viability of a dry excavation remedial construction can be determined. The two most critical scenarios are the potential development of a basal heave and/or bottom uplift instability of the dewatered excavation bottom surface during the construction work. However, the development of any one of the five potential instability scenarios would represent a serious health and safety risk to construction personnel working within the dry excavation footprint. Completion of quantitative analyses that show a high probability of occurrence of any of these scenarios would represent an unsafe and therefore unacceptable condition, and would be sufficient reason for WESTON to recommend to the United States Environmental Protection Agency (USEPA) that the dry excavation remedial action be rejected as a technically feasible site cleanup option.

WESTON completed detailed geotechnical analyses to quantitatively evaluate the five potential geotechnical instability scenarios relevant to the completion of a dry excavation remedial action at the project site. The five analyses included:

1) Design of the cantilevered sheet piling retaining structure.

2) Excavation Bottom Basal Heave Instability Potential.

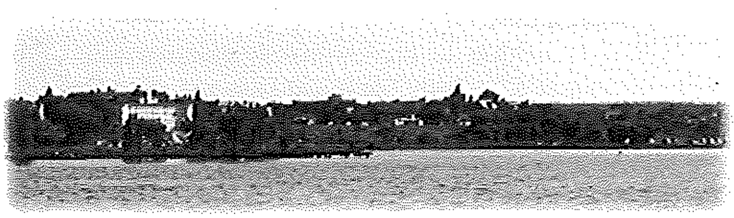
3) Excavation Bottom Uplift Instability Potential.

4) Exit Gradient (or liquefaction) Instability Potential.

5) Rotational Instability Potential.

It is important to note that, as appropriate and necessary, these analyses incorporated the destabilizing effects of the artesian pore water pressure within the underlying CFF aquifer on both the stability of the dewatered excavation bottom and the cantilevered steel sheet piling retaining structure which is a necessary component of this construction. The results of the five technical analyses are presented and discussed in detail in the Technical Report and Technical Analysis included with this technical submittal.

An additional instability concern has been raised in previous documents prepared by others, but was not quantitatively evaluated in these documents. This concern involves whether the presence of fractures or sand seams within the MCF soil stratum should be the basis for a



reduction of the shear strength of these soils from values determined from the field and laboratory testing programs. Our review of the collected field and laboratory test data, as well as a visual examination of the split spoon and Shelby Tube soil samples recovered during the field work, has led WESTON to conclude that these concerns are insignificant based on the subsurface conditions encountered at the Site. We further believe that the possible presence of these anomalies, and their negative impact on the shear strength of the MCF soils in the various instability scenarios evaluated in this study, is easily accounted for by the inherent conservatism required by the standards of the geotechnical engineering profession in completing these quantitative evaluations.

It must be noted that it is common, especially when working in wet environments, to experience localized non-critical manifestations of what may be considered quasi- bottom uplift or exit gradient instabilities where the excavation bottom may deflect slightly upward (on the order of several inches) or soil pore water from the top of the MCF stratum may seep into the open, dewatered excavation bottom. These developments are non-serious and commonplace in excavation work with plastic materials such as clayey silts and clays. These occurrences are typically and easily remediated by staged excavation and grading activities and/or active dewatering of the excavation bottom using bottom suction pumps.

WESTON TECHNICAL DISCUSSIONS

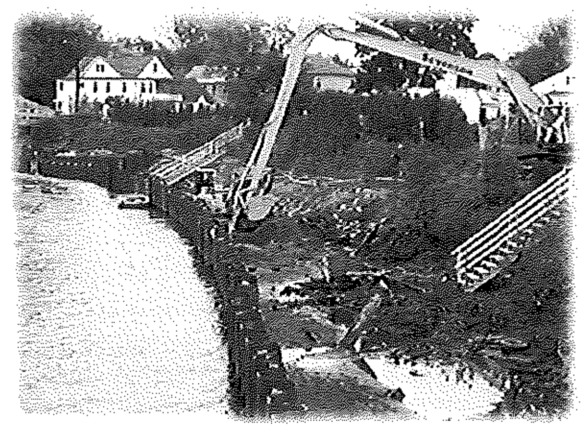
Two Technical Discussions were prepared by WESTON to address the excavation bottom basal heave and excavation bottom uplift instability potential of the proposed dry excavation remedial action. These were authored by Dr. William L. Deutsch Jr., Ph.D., P.E., a Geotechnical Engineering Consultant under subcontract to WESTON, and are included with the Technical Submittal. Each technical discussion was prepared as a detailed, high-level, state-of-practice technical "guidance document" to be used to quantitatively assess both basal heave and bottom uplift instability potential via the calculation of a Factor of Safety against development of the instability scenario. The technical discussions were developed consistent with the encountered subsurface environment at the project site, and include the destabilizing effects represented by the presence of artesian pore water pressure within the predominantly granular soils of the CFF. Both Technical Discussions were thoroughly reviewed and critiqued by Dr. Joseph Schulenberg, Ph.D., P.E., a Regional Technical Specialist with the Lakes and Rivers Division of the United States Army

Corps of Engineers Chicago District, and modified consistent with his prepared written commentary before finalizing the documents in the form presented with this Technical Submittal.

THE FACTOR OF SAFETY AS USED IN ENGINEERING ANALYSES

An engineering concept referred to as a Factor of Safety (FS) is used to quantify the probability of occurrence of a potential instability scenario. A Factor of Safety is the ratio between stabilizing or resisting forces in the numerator of the expression to destabilizing or driving forces in the denominator of the expression. A calculated FS value that is greater than 1.0 indicates that the stabilizing forces are greater than the destabilizing forces. Therefore, when a calculated FS value is greater than 1.0, the potential instability scenario quantified by the calculated FS value should theoretically not occur. The probability of occurrence for an instability scenario represented by a calculated FS value that is greater than 1.0[2] decreases as the difference between the calculated FS value and an FS value of 1.0 increases.

Consistent with this relationship, the minimum required FS values for various potential instability scenarios are generally established at values that are numerically higher, sometimes significantly higher depending on the consequences of the instability occurring, than the value of 1.0 in accordance with standards of the engineering profession. For example, standard geotechnical engineering practice requires that the calculated FS values for the basal heave and bottom uplift instability scenarios determined to be the most serious of the technical concerns relevant to this project must exceed minimum required values of 1.50 and 1.25, respectively.



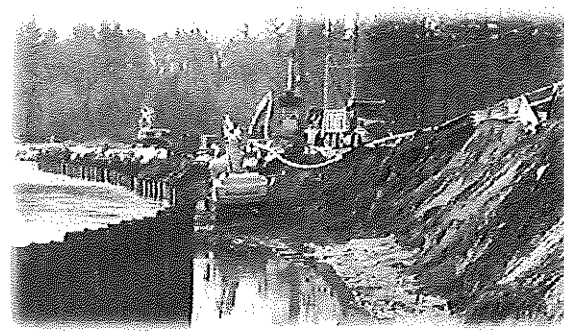
RESULTS AND CONCLUSIONS OF WESTON'S TECHNICAL ANALYSES

WESTON completed the five relevant and essential geotechnical engineering technical analyses noted above to determine the technical feasibility of a dry excavation remedial action at the Ashland project site. Based on the results of these analyses, WESTON has concluded that acceptable FS values that were numerically higher, sometimes significantly, than the minimum required values based on standard geotechnical engineering practice, were calculated for all five instability scenarios. Consistent with these results, WESTON has therefore concluded that the subsurface environment at the Ashland Lakefront

[2] A calculated FS value of 1.0 for a given instability scenario theoretically indicates that the stabilizing forces are exactly equal to the destabilizing forces and, therefore, represents the incipient or impending failure condition for this scenario.

Superfund Site is geologically suitable and favorable for a future dry dredging remedial action project which can be safely completed in the field using standard engineering and construction practices, and will be protective of both human health and the environment.

The analyses completed by WESTON initially assumed a full excavation footprint approach in which the entire near shore excavation area (7.5 acres ±) would be sheeted, dewatered, excavated and backfilled as one large cell. However, it is common practice for projects of this nature to instead be completed by creating smaller sheeted cells within the larger sheeted footprint using internal lines of sheet piling which structurally connect to the perimeter sheeting. This remediation concept permits a much more efficient and cost effective staged construction approach to be used in which one smaller cell at a time can be remediated. In this regard, WESTON's completed technical analyses have also demonstrated that, as the area of a sheeted dry excavation cell is reduced, the FS against the excavation bottom uplift instability scenario increases, while the FS values for the remaining four instability scenarios remain the same. Therefore, WESTON has also concluded that completing the dry excavation remedial action at the project site using a to-be-determined number of smaller sheeted cells rather than one large cell is also technically feasible and can be safely completed in the field using standard engineering and construction practices, and will be protective of both human health and the environment.



WESTON'S RELEVANT Q&E

WESTON has successfully and safely designed and installed steel sheet piling as temporary retaining structures on numerous projects that involved the removal of deeply buried contaminants. Other sheet piling design/construction projects completed by WESTON were used to permit vertically faced excavations to be completed to great depths for the purpose of constructing a new below ground structure such as a buried pipeline or underground storage tank. Many of these projects were completed in open water bodies, including rivers, canals, and lakes in which larger differential water levels existed than will be experienced at the Ashland Lakefront Superfund Site. The photographs included in this Executive Summary depict one of those projects.

TECHNICAL REPORT

I. Purpose and Objective

Weston Solutions Inc. (WESTON®) was contracted by the United States Environmental Protection Agency (USEPA) to re-evaluate a preliminary technical analysis completed by WESTON in 2009 that assessed the feasibility of utilizing dry dredging (aka "dry excavation") technology and construction procedures to remove contaminated offshore sediments and wood waste at the Ashland Lakefront Superfund site (Site). Our 2009 analysis was completed using a limited database of onshore soil borings and offshore sediment samples available at that time. The revised Technical Analysis (Analysis) discussed herein, utilizes the database obtained from a recently completed offshore field investigation by Anchor QEA as presented in their report entitled "Offshore Sampling Data Report, Ashland Lakefront Superfund Site" dated November, 2013 (Anchor Data Report). The offshore geotechnical investigation performed by Anchor consisted of numerous soil borings and cone penetrometer soundings that were accompanied by a variety of field and laboratory tests to further define and quantify the physical properties and shear strength parameters of the encountered subsurface soil strata which underlie the project site as interpreted in the field by Anchor personnel. The results of Weston's Technical Analyses are summarized in this Technical Report and are discussed in detail in the nine (9) Technical Exhibits presented as Appendix A of the Technical Submittal.

As part of the Analysis, WESTON and consulting engineer Dr. William L. Deutsch Jr., Ph.D., P.E. authored two peer-reviewed Technical Discussions that present the current engineering practice and procedures for analyzing basal heave and bottom uplift potential instability scenarios that must be considered when designing a dry excavation in an aqueous environment. These Technical Discussions represent a compendium of current standard engineering practice regarding the quantitative evaluation of each scenario, and were structured as guidance documents with step-by-step analysis procedures for calculating the Factor of Safety against the potential development of each instability scenario. A peer review of the two documents was provided by Dr. Joseph Schulenberg, Ph.D., P.E., a Regional Technical Specialist with the Lakes and Rivers Division of the United States Army Corps of Engineers Chicago District. Preliminary drafts of both documents were subsequently revised consistent with Dr. Schulenberg's submitted review comments. The final versions of the Technical Discussions presented herein have also been peer reviewed and approved by Dr. Schulenberg. These are included as Appendices B and C of this Technical Submittal.

During preparation of the analysis and discussions, WESTON also reviewed and critiqued the results presented in the "Shoreline and Offshore Geotechnical Evaluation Report, Ashland Lakefront Superfund Site" authored by Anchor QEA in December 2013 (Anchor Evaluation).

Numerous and serious technical deficiencies, inaccuracies, inconsistencies and overly conservative assumptions were evident based on our review of this document. If necessary, our comments based on this review will be provided under separate cover and therefore are not discussed in this Technical Report except when necessary.

II. Professional Background

WESTON employs a team of civil and geotechnical engineering professionals whose involvement with the geotechnical aspects of the Ashland Lakefront Project is led by Adam Brown, P.E. Mr. Brown has 10 years of professional experience in soils/geotechnical engineering analysis, design and construction and has an earned Master's degree in geotechnical engineering. Mr. Brown has performed numerous geotechnical evaluations and designs; field investigations for geotechnical, hydrological, and environmental analysis and designs; and technical analysis for geotechnical modeling of slope stability, settlement, and foundation capacities for infrastructure upgrades and remediation projects at commercial, public, and industrial sites for state, federal, and private clients. Mr. Brown has performed construction oversight and field engineering services for many geotechnical projects involving landfill closures, sheet pile walls, anchoring technologies, excavation support, deep and shallow foundations, sediment dredging, shoreline bulkhead walls, soil removal actions, seepage, and on-site treatment facilities. Mr. Brown is a registered Professional Engineer in the State of Pennsylvania.

Dr. William L Deutsch Jr., Ph.D., P.E. is a private consulting engineer under subcontract to WESTON. Dr. Deutsch assists the Geotechnical team at WESTON with complex geotechnical engineering designs. He additionally helps educate project team members, WESTON clients, regulators and the public regarding the technical aspects of geotechnical engineering projects through public and private forums and information sessions. He has more than 40 years of consulting experience in geotechnical earth support and foundation engineering, geosynthetics and geo-environmental engineering analysis/design as well as hydrology, hydrogeology, and hydraulics engineering. Dr. Deutsch has extensive experience in conceptual to final design engineering, project and construction management; preparation of feasibility studies, conceptual and final design studies, reports, construction-level drawings and technical specifications; completion of forensic technical studies/reports and expert witness testimony; and client and regulatory liaison. Dr. Deutsch has also served as an adjunct professor at Drexel University where he taught graduate courses for 16 years in the subjects of retaining structures, soil deformation, rock mechanics, soil stabilization, soil behavior, slope stability, and shallow and deep foundation design. Dr. Deutsch is a registered Professional Engineer in the State of Pennsylvania.

Professional resumes and key project write-ups for Mr. Brown and Dr. Deutsch are included in Appendix D of this Technical Submittal.

III. Site Background

The Ashland Lakefront Superfund Site is located in the town of Ashland, Wisconsin, along the southeastern banks of Chequamegon Bay of Lake Superior and encompasses a recreation area known as Kreher Park, a bluff upland from the park, a former waste water treatment facility (now demolished), and an approximately 16 acre offshore area within Chequamegon Bay. All of these areas have experienced extensive contamination as typically produced by the by-products of the manufacturing of heating and lighting gas, i.e., hydrocarbons and heavy tars. The focus of the Technical Analysis discussed herein is the offshore portion of the Site within a 200 foot offset distance from the shoreline.

The local offshore subsurface environment consists of contaminated fill materials consisting primarily of wood waste and contaminated offshore soft sediments which directly underlie the free water of the bay. These materials are underlain by the very low permeability soils of the Miller Creek Formation (MCF). These soils consist of well-defined and consistent layers of hard silts and soft to medium stiff clays. The MCF soils overlie the Copper Falls Formation (CFF), a highly permeable soil stratum that contains dense sands and gravels that extend to great depth. As discussed below, the CFF is also a regional artesian aquifer.

The offshore sediments and wood waste generally decrease in thickness with increasing distance from the Site shoreline, from an approximate 14-foot thickness inland from the shoreline to an approximate 4 foot thickness near the 200 foot offshore removal extents. The free water depth of the bay increases in roughly the reverse manner with increasing distance from the shoreline. The underlying MCF tends to increase considerably in thickness with increasing distance from the shoreline, from approximately 25 feet at the shoreline to over 65 feet near the proposed 200 foot offshore removal extents.

The CFF at the Site and surrounding area is a regional artesian aquifer within which artesian pore water pressures exist. An artesian pore water pressure condition occurs when the pressure within the aquifer soils is sufficient to force the water to an equilibrium level that is higher than the existing ground surface. The artesian head at any land or offshore location can easily be measured by vertically extending the casing of a well screened within the aquifer to a top elevation such that the pressurized water comes to an equilibrium level within the casing. The artesian head is then determined as the vertical distance between the ground surface on land, or the free water surface of the bay for offshore locations, and the higher equilibrium water level within the casing. The maximum measured value of the artesian head acting at the bottom of the

MCF was measured as approximately 12 feet within the dry dredge footprint based on data obtained from the recently completed offshore field work. To provide perspective to the severity of this condition, if one were to apply the same logic to a potable water utility line that is servicing an average American home, the water level in an open vertical standpipe connected to the pressurized line would extend to over 135 feet above the pipe; that is, to a level more than 10 times that produced by the CFF artesian pressures.

Artesian pressures within aquifers are a very common phenomenon throughout northern Wisconsin[1]. This condition develops and exists for a variety of geologic reasons. The natural manifestation of the artesian pressure within the CFF can be seen throughout the region in the form of natural springs which freely emit unimpeded and continuous water flow. In addition, groundwater from wells that are screened within the CFF with a top-of-casing elevation near the existing adjacent ground surface flows freely without pumping.

IV. Site Remediation Background

The USEPA and the Wisconsin Department of Natural Resources (WDNR) have performed numerous site investigations at the Site since 1991. Among the many findings developed from these studies, the regulatory agencies discovered that Northern States Power of Wisconsin (NSPW), a subsidiary of Xcel Energy (Xcel), had potentially contributed to the contamination of soils and groundwater underlying the Site through the operation of a manufactured gas plant. In September 2010, the USEPA issued a Record of Decision (ROD) which defined the preferred technologies for the remedial action to be implemented at the Site. A hybrid remedy was selected for the offshore portion of the Site. The hybrid approach included excavation of the near shore contaminated bay bottom sediments and miscellaneous (primarily wood) materials located within an approximate 200 foot offset distance from the shoreline in a dry condition (i.e., dry dredging). This would be facilitated by the initial installation of an impermeable vertical structural barrier (i.e., cantilevered steel sheet piling) to be placed around the perimeter of this footprint. This would be followed by the dewatering of the footprint internal to the sheet piling and the subsequent excavation of contaminated bay bottom materials "in the dry". This component of the hybrid remedy was selected because the sediments located within this offset distance are generally very thick and contain higher quantities and concentrations of the contaminants of

[1] William J. Drescher. (1956). *Information Circular Number 3 - Ground Water in Wisconsin.* Madison: University of Wisconsin.

concern (COCs), NAPL[2], and free product than the contaminated sediment deposits further offshore. In addition, these near shore contaminated materials exist beneath a layer of wood waste. These factors rendered wet dredging as a means of removing these materials technically difficult and potentially impractical in this area. The second component of the hybrid remedy presented in the ROD specified that contaminated sediments located beyond the 200-foot offset distance be removed using wet dredging technology, citing thinner layers of contaminated bay bottom sediments and the absence of wood debris, thereby rendering wet dredging technology technically feasible at this location.

V. Dredging Technology Background

Removal of the offshore contaminated bay sediments can largely be accomplished using two construction techniques: dry dredging and wet dredging. Dry dredging involves dewatering the excavation footprint and using conventional earth moving equipment to remove the targeted sediment. Wet dredging is usually subdivided into mechanical and hydraulic categories. Mechanical wet dredging can be performed with special excavation buckets and clamshell dredging cranes. Hydraulic dredging usually involves specialized equipment that excavates and transports sediment and water in pipelines by means of large pumps and suction equipment. Both dry dredging and wet mechanical methods employ the use of an excavator or other large mechanically operated bucket to remove the sediments with the simple difference that one is performed with the water removed from the dredge area and the other with the water in place, hence, the designations dry and wet dredging, respectively.

One of the major concerns of the use of wet dredging technology at the Site is that some portion of the targeted sediment is inevitably left behind as an inherent limitation of the technology. Excavating in water will generate some level of sediment re-suspension and unexcavated residuals. Re-suspended sediment is the byproduct of both the disturbance of the sediment and the inevitable loss of sediment from excavation buckets or clamshells as the bucket is lifted through the free water column.

At sites where dredging is proposed for the removal of contaminated sediment, the issues of resuspension, residuals, and potential off-site losses of COCs is of concern to the project designers, environmental resource managers, regulators and the public. Resuspension and dredge residuals can leave behind contaminants that are intended for removal, and off site migration of

[2] NAPL is an acronym used for a non-aqueous phase liquid. This is a liquid that does not readily mix or dissolve in water such as solvents and petroleum products. NAPLs can be lighter or heavier than water causing them to float or sink.

the contaminants can pose a risk to other areas of the water body. This risk is magnified as the volume of dredge material increases.

An example that illustrates re-suspension with respect to the concern at the Ashland Site with wet dredging is a container of Italian salad dressing consisting of olive oil, vinegar, and herbs. The herbs that are suspended in the container can be visualized as sediment particles and the olive oil as the NAPL and other COCs.

.In this example, if the mixture is allowed to sit for a sufficient period of time, essentially all of the herbs (suspended sediment) will settle to the bottom of the glass, and the oil will separate itself from the vinegar (bay water). An alternative to wet dredging is dry dredging. Dry dredging technology typically employs the use of steel sheet piling or an equivalent retaining structure to physically separate the area to be dry dredged from adjacent areas of free water. This permits the dry dredge footprint to be dewatered and exposes the underlying contaminated sediments allowing them to be conventionally excavated. Dry dredging typically does not produce resuspension of sediment given the absence of a water column within the excavation. Residuals can be managed with carefully executed and monitored excavation techniques. Off-site migration of contaminants are also easier to control than with wet dredging. A negative aspect of dry dredging is that, unlike wet dredging, volatilization of submerged contaminants could occur following dewatering of the project site. This in turn could create noxious or otherwise undesirable odors which could be released to the atmosphere should this chemical process occur. Odor control measures such as the direct application of foaming agents to the surface of contaminated materials are available which may be easily implemented to suppress these odors.

The dry dredging alternative specified in the ROD would require the initial installation of a cantilevered sheet pile retaining structure around the perimeter of the dry dredging footprint to isolate the contaminated bay bottom sediments. Free water within this enclosed area would be removed, and the contaminated sediment, oil-like NAPL, and comingled wood waste would be removed from the dewatered bay bottom using conventional excavation equipment. The dry dredging alternative would be especially effective in preventing the outward migration of contaminants to deeper water in the bay. The dry dredging approach would also permit effective removal of the NAPL which is very difficult to achieve using wet dredging technology.

VI. Standard Engineering Practice

Geotechnical engineers are responsible for providing technically sound and feasible solutions for projects which involve the subsurface environment. Examples of these include designing proper foundation support for structures, general earthworks projects such as the design of earthen embankments for highway support, or remediation projects such as dry dredging at the proposed

Ashland Site. Regardless of the task, the primary concern of a geotechnical engineer must be both the health and safety of those directly involved in the field work during completion of the project and the long term health and safety of the public following completion of the project. An equally important concern is that the project design and subsequent construction be generally protective of the environment. These general concerns need to be balanced by site-specific concerns such as the project goals and regulatory mandates. It is therefore of utmost importance when selecting design or analysis procedures for use in evaluating potential geotechnical instability scenarios relevant to a project that the most up-to-date and technically relevant of the available quantitative analysis procedures as well as other guidance documents be identified through a literature search. Quantitative procedures and guidance documents should then be subsequently critiqued regarding the assumptions on which the design or analysis procedures were developed, and modified if necessary consistent with the actual site and subsurface conditions applicable to the project before selecting the procedures to be used in completing the relevant instability potential analyses.

Geotechnical engineers are often faced with the decision of selecting the most relevant and appropriate instability potential analysis from, typically, more than one existing theory and/or practice. In general, some of these theories and practices have been in existence for many decades while others have been more recently developed consistent with the latest highly sophisticated computer-based finite-element modeling techniques. Other instability potential analysis procedures are simply based on guidance developed from decades of real-life project experiences or from technical research completed by practicing engineers or academicians. These guidance documents are typically referred to as *common engineering* practices or *industry standards* in the engineering profession.

In the early phases of a project, it is commonplace in engineering practice to use limited datasets and simplifying conservative assumptions to make broad decisions on how to proceed with a quantifiable analysis or design. As the project develops with additional constraints, scope, and environmental factors becoming relevant, a more definitive analysis approach may be developed that is representative of the known site and subsurface conditions. It is with this mindset that WESTON prepared two detailed Technical Discussions that address the most serious of the potential instability scenarios relevant to the dewatered excavation bottom during completion of a dry dredge remedial action at the Site: bottom uplift and basal heave. These discussions were developed consistent with currently available project data as presented in the Anchor QEA Data Report as well as the generalized subsurface environment encountered at the project Site. The Technical Discussions also incorporate the destabilizing effects of the artesian pore water pressure present within the predominantly granular soils of the CFF.

VII. WESTON Technical Discussions and Analysis

The two Technical Discussions were authored by Dr. William L. Deutsch Jr., Ph.D., P.E. Each discussion is a high-level, state-of-practice technical guidance document developed consistent with the encountered subsurface environment at the Site. The discussions begin with a presentation of the underlying theories, governing equations, and assumptions applicable to each of the two potential instability scenarios. They then introduce more recent modifications and upgrades to the basic theories that have been developed and presented in the geotechnical literature over time. Both Technical Discussions conclude with the presentation of a recommended equation for calculating the Factor of Safety (FS) against the development of the relevant instability scenario, as well as a recommended step-by-step analysis procedure for sequentially calculating the parameters necessary to determine the FS value from the governing equation. The destabilizing effects of the artesian pressures that exist in the CFF were also "built in" to both Technical Discussions. The Technical Discussions were thoroughly reviewed by Dr. Joseph Schulenberg, Ph.D., P.E., a Regional Technical Specialist with the Lakes and Rivers Division of the United States Army Corps of Engineers, Chicago District, and modified consistent with his prepared written commentary before finalizing the documents in the form presented in this Technical Submittal. Dr. Schulenberg found both Technical Discussions to be consistent with typical geotechnical engineering practice, as well as conservative with respect to the assumptions made and the developed governing equations and recommended analysis procedures. Dr. Schulenberg also found both Technical Discussions to be acceptable for use in evaluating the excavation basal heave and bottom uplift instability potential of the proposed dry dredging remedial action at the Site.

WESTON geotechnical engineers also prepared a Technical Analysis that consists of nine exhibits that each contain parameter derivations and calculations based on the subsurface profile encountered within the two most critical test boring locations identified by the Anchor Evaluation that are directly relevant to the dry dredge excavation footprint. These subsurface profiles include offshore test borings AQ-SB-02 and AQ-SB-04 where the highest potential for bottom uplift and basal heave instability, respectively, were calculated by Anchor. In addition, similar analyses were also completed for test boring SB-185 completed by Foth-Environcon at an inland location near the shoreline of Chequamegon Bay in December, 2012. The Anchor report also indicated that the subsurface conditions at this shoreline test boring location yielded a very low, unacceptable FS value against the development of a potential bottom uplift instability should a dry excavation remedial action be completed in this area of the project Site.

WESTON's Technical Analysis evaluates five potential instability scenarios relevant to a dry dredge excavation using a steel sheet piling retaining structure embedded in a low permeability stratum (MCF) to separate the dry dredging footprint from the free water of Chequamegon Bay that exists on the outside of the retaining structure. The analyses also consider the destabilizing effects of the underlying artesian CFF aquifer. The five analyses include the following:

1) Design of the cantilevered sheet piling retaining structure. Proper design of this structure includes determination of its embedment depth below the finished excavation bottom to ensure that the structure will remain stable and vertical for the duration of the dry dredging work. The sheet pile retaining structure should not lean inward excessively over time since this occurrence could result in the rapid flooding of the dry dredging footprint if the waters of the Chequamegon Bay overtop the leaning structure. The design also requires proper selection of a sufficiently heavy sheet pile section such that it will not excessively bend due to the unbalanced forces acting upon it. In addition, a sufficiently heavy section is necessary to resist bending and other damage from driving stresses during installation of the sheets.

2) Basal heave instability potential exists when the removal of soil and water on the excavation side of the sheet pile retaining structure creates a weight imbalance whereby the soil and water weight retained by the structure on the outside of the sheeting is sufficiently heavy to create a shear failure within the soil mass beneath the retaining structure. Should this occur, the sheared failure mass would rotate inward and cause a severe uplifting of the excavation bottom soils in front of the sheeting. This occurrence would in turn destabilize the sheet piling by causing it to rotate inward. Under excessive rotation and concurrent leaning of the retaining structure, there exists a potential to quickly flood the dry excavation footprint should the free water of the bay overtop the leaning structure.

3) Bottom uplift instability potential exists when sheet piling intended to facilitate a future excavation is driven partially into, or completely through, a low permeability confining soil layer (MCF) that is underlain by a higher permeability granular soil layer (CFF) whose pore water pressures would cause the water level in a wellpoint screened in the granular soils to rise above the elevation of the excavation bottom. The greater the elevation of the water level above the excavation bottom, the greater the risk of excavation bottom uplift instability. In addition, this concern is further exacerbated by the artesian pore water pressure which exists within the granular soils of the CFF. The artesian pressure acts to uplift the MCF. Should this uplift force be insufficiently counter balanced by the downward acting resisting forces generated by the weight and internal shear strength of the uplifted soils, a potentially destabilized excavation bottom could result.

4) An exit gradient instability potential exists when the water levels on the inside and outside of the sheet piling are different. In this instance, upward seepage flow develops in the soils immediately adjacent to the inside face of the sheet piling. If sufficiently high, the exit

gradient associated with this flow could potentially destabilize the sheets by "liquefying" these soils.

5) A rotational instability potential exists as a result of a soil/water weight imbalance between the inside and outside of the sheet piling. This potential instability scenario is similar to that which would occur in a basal heave failure.

An additional instability potential has been raised in previous documents prepared by others, but has not been quantitatively evaluated in these previous reports. This potential involves whether the presence of fractures or sand seams within the MCF soil stratum should be the basis for a reduction of the strength of the MCF soils from values determined from the field and laboratory data, as well as precipitate a massive and sudden soil structure failure through crack propagation within the MCF stratum. Our review of the collected field and laboratory test data, as well as a visual examination of the split spoon and Shelby Tube soil samples recovered during the field work has led WESTON to conclude that these concerns have no merit based on the subsurface conditions encountered at the site. We further believe that the possible presence of these anomalies, and their negative impact on the shear strength of the MCF soils in the various instability potential scenarios evaluated in this study, are of little concern and accounted for in the inherent conservatism required by the standards of the geotechnical engineering profession in completing these quantitative evaluations.

VIII. Technical Analysis Summary

The attached Technical Analysis presents the results of the five potential instability scenarios relevant and essential to determining the technical feasibility of a dry excavation remedial construction at the Site in the form of nine Technical Exhibits. Exhibit 1 discusses the offshore subsurface investigation completed in the Summer and Fall of 2013 as well as the generalized subsurface conditions within the proposed dry excavation footprint developed from this data base. Exhibit 2 discusses the selection of required geometric and geotechnical input parameters necessary to complete the five relevant and essential geotechnical instability scenarios discussed above, and in Exhibit 3, which could potentially develop during the course of a dry excavation remedial action at the Site. Exhibits 4 through 8 discuss in detail the five relevant and essential geotechnical instability scenarios. The ninth and final exhibit discusses and presents the Results and Conclusions of the Technical Analyses. A brief discussion of the exhibits, their relation to the five instability scenarios, and the standard geotechnical engineering practices employed, is presented in the following sections.

A. Subsurface Conditions – Exhibits 1 and 2

Subsurface conditions within the proposed offshore dry excavation footprint were investigated by Anchor QEA and presented in the Anchor Data Report dated November 2013. A total of 7 test borings designated as AQ-SB-01 through AQ-SB-07A, and 14 cone penetration tests (CPTs) designated as AQ-CPT-01 through AQ-CPT-14N were completed. At a four locations, a test boring was co-located with a CPT completed within a short distance of the boring. The test borings were completed by first advancing a 24-inch-long split spoon or thin walled Shelby Tube sampler through the subsurface environment for the purpose of collecting samples for field and laboratory testing to evaluate the physical and engineering properties of the soil. The cone penetrometer test consisted of advancing a piezocone[3] through the subsurface environment and recording strength and porewater pressures in situ. Formalized test boring and CPT logs, as well as the laboratory test data, were presented in the Anchor Data Report. The field and laboratory data from this report formed the basis for the technical discussions and analyses presented in Exhibits 1 and 2.

An indirect measurement of the shear strength and density condition of encountered subsurface soils was obtained via the completion of split spoon sampling of these soils using the procedures of ASTM D-1586, the Standard Penetration Test (SPT), at each of the test boring locations. "Blow counts" measured in blows per foot (bpf) during this test can be converted to Standard Penetration Resistance (SPR) or "N" values that are presented on the boring logs. Field measurements were also completed to determine the undrained cohesive shear strength of encountered clayey soils using a pocket penetrometer (PP), a pocket Torvane® (TV), and an in situ vane shear test (VST). In addition, thin walled Shelby Tube samples of soft, compressible clayey soils were collected for subsequent shear strength, permeability, and compressibility testing in the laboratory. Both Shelby Tube and split spoon soil samples were also used to complete laboratory physical property testing on the encountered soils.

Each CPT sounding includes a continuous readout of the direct measurement of the in situ tip resistance and sleeve friction that may be used to calculate soil engineering properties, including undrained cohesive strength, at each CPT location. At various depths within the soundings, a direct measurement of the in situ pore water pressure was directly recorded. This measurement was attempted in each of the CPT soundings at the observed MCF/CFF interface to directly

[3] A piezocone is a device that measures downhole soil load responses and pore water pressures through various electronic sensors attached to the piezocone. The device is pushed, rather than driven, through the encountered soil strata using down pressure from the cone rig.

measure the artesian pore water pressure at this location. Pore pressure measurements were additionally collected at points deeper within the CFF and sporadically within the MCF.

Each of the test borings and CPT soundings encountered the same three basic geological formations as discussed previously. Although the soils of each formation were reasonably consistent, geologically speaking, both in strength and physical properties, their thicknesses did vary across the Site as described in detail in Exhibit 1.

Development of a representative cross section of the encountered subsurface environment within the dry excavation footprint is necessary input to the geotechnical analysis of the five potential instability scenarios. Exhibit 2 of the Technical Analysis describes in detail how the measured subsurface data presented in the Anchor Data Report is used by WESTON to develop representative cross sections for the two most critical test boring locations identified by Anchor (i.e. offshore borings AQ-SB-02 and AQ-SB-04).

B. Structural Stability of Cantilevered Steel Sheet Piling Wall – Exhibit 4

A cantilevered sheet piling design for the critical 200 foot offshore sheet pile wall was completed using the software package ProSheet developed by Skyline Steel Company. For given input data relevant to both the stratigraphic profile and the physical and engineering properties of encountered soil strata within the profile, the software package provides output results that permit determination of the minimum required embedment depth of the sheets, the minimum required section modulus[4] of an acceptable sheeting section, and an estimate of the maximum deflection of the steel sheets which occurs at the top of the wall for cantilevered sheet piling.

To complete the ProSheet analysis, WESTON made a number of assumptions in order to accurately represent the subsurface conditions at the Site. Some of these assumptions were also made to increase the level of conservatism in the design. These assumptions include the following:

1. Drained cohesive soil shear strength parameters assuming normally consolidated soil behavior (i.e., $c_u' = 0$, $\phi' > 0°$) were used to calculate the active earth pressures of the clayey soils of the MCF on the retained side of the sheeting. This assumption produces significantly higher, and therefore more conservative, driving lateral earth pressure forces which act to destabilize the wall than the use of undrained soil shear strength parameters

[4] The section modulus is a physical property of sheet piling that is proportional to the cross-sectional area of the sheets. A thicker sheet will have a higher section modulus than a thinner sheet, all else equal. The higher the Section Modulus of a sheet piling section, the greater the section's resistance to bending stresses developed within the section as a result of its use as a retaining structure.

that more accurately represent the short term shear strength behavior of these soils which are appropriate for use when designing a temporary retaining structure.

2. The water table on the excavated side of the wall was set at a depth of 2 feet below an assumed 2-foot overexcavation depth of the MCF soils; that is, at a depth of 4 feet below the surface of the MCF soils. This assumption is consistent with the potential need to overexcavate the top several feet of the MCF soils due to the possible presence of contaminants within these materials and the potential need to dewater the MCF soils by pumping from passive drainage trenches prior to excavation of these soils.

3. The ProSheet analysis incorporated the effects of both the artesian pore water pressure within the underlying CFF soils and the unbalanced hydrostatic head created by differing water surface levels on the retained and excavated sides of the wall.

4. As is common in geotechnical engineering practice, the effect of upward seepage flow through the MCF soils on the excavated side of the sheet piling wall was appropriately modelled in the sheet piling design by reducing the unit weight of the soils in the passive pressure zone of the stratigraphic profile. This reduction in unit weight was calculated consistent with the estimated largest seepage gradient expected during the dry dredge construction activities.

The impact loadings from external forces due to wind-generated waves is necessary input to the design of the critical 200-foot offshore sheet piling retaining structure. Appendix A of Exhibit 4 documents the calculation of the estimated breaking and non-breaking wave forces relevant to the sheet piling design. The forces were subsequently incorporated into the ProSheet analyses.

A major consideration when designing a sheet piling retaining structure is the issue of drivability of the sheets; in this instance, within and through the very stiff to hard clayey silt (ML) soils that were encountered at the top of the MCF stratum. This issue was reviewed and discussed with a Geotechnical Engineer from Skyline Steel Company (a subsidiary of Nucor) as discussed in Exhibit 4. Based on these discussions, it was determined that sheet pile section AZ-26 or heavier would permit routine installation of the sheets and be suitable for use at the Site. In this regard, it is noted that Section AZ-26 sheets are commonly used and readily available for both purchase as permanent walls and rental for temporary "drive and pull" walls. The term "drive and pull" refers to a project in which steel sheet piling is installed (i.e., driven) into the ground for a defined duration during which the structure permits a given construction to be completed; i.e., in this instance, the excavation of contaminated bay bottom sediments in the dry.

The sheet piling is subsequently removed (i.e., pulled out) from the ground as one of the final components of the project.

C. Basal Heave Instability Potential– Exhibit 5

WESTON has prepared a detailed and technically advanced quantitative analysis procedure for evaluating the potential for a basal heave instability to develop within the MCF soils that underlie and surround the embedded cantilevered steel sheet piling retaining structure whose installation is an essential component of the dry excavation remedial construction. Exhibit 5 briefly discusses the Technical Discussion entitled, "Development of Recommended Quantitative Analysis Procedure for Evaluating Basal Heave Instability Potential of a Dry Excavation Footprint at the Ashland Lakefront Site," which was prepared by Dr. William L. Deutsch, Ph.D., P.E. and is included as Appendix B of this Technical Submittal. The analysis is based on Terzaghi's original theory and was structured to permit basal heave instability potential to be quantitatively evaluated via the calculation of a Factor of Safety against development of this instability.

Basal heave instability potential theory was initially developed by Terzaghi in 1943 for shallow sheeted excavation depths in wide footprint excavations similar to that proposed for the Ashland Site dry excavation remediation[5]. Major modifications and refinements to Terzaghi's theory that have been presented in the geotechnical literature since 1943 were incorporated into the Basal Heave Technical Discussion.

D. Bottom Uplift Instability Potential– Exhibit 6

WESTON has also prepared a detailed and technically advanced quantitative analysis procedure for evaluating the potential for a bottom uplift instability to develop within the MCF soils due to the artesian uplift pore water pressure that exists within the underlying CFF aquifer. This Technical Discussion entitled, "Development of Recommended Quantitative Analysis Procedure for Evaluating Excavation Bottom Uplift Potential of a Dry Excavation Footprint at the Ashland Lakefront Site," was also prepared by Dr. William L. Deutsch, Jr., Ph.D., P.E. and is included as Appendix C of this Technical Submittal. The analysis was structured to permit bottom uplift instability potential to be quantitatively evaluated via the calculation of a Factor of Safety against development of this instability.

[5] As also discussed in the Technical Discussion, this is in sharp contrast to an alternate theory for evaluating basal heave instability potential developed by Bjerrum and Eide in 1956 for deep sheeted excavation depths in narrow footprint excavations such as those necessary to install a deep buried pipeline.

Bottom uplift instability potential theory was initially developed by assuming that the ratio of the downward acting weight of the uplifted failure mass to the upward acting force due to the artesian pore water pressure within the CFF quantified the instability potential. This approach has repeatedly been noted as being conservative due to its omission of the shear strength along the vertical sidewalls of the uplifted failure mass, which would realistically mobilize as additional uplift resistance. Older references in the geotechnical literature qualitatively noted both the reality of this shear resistance and its omission in discussions of this issue presented in these references. More recent discussions of this issue, most notably Koutsoftas in his peer-reviewed technical paper entitled, "State of Practice: Excavations in Soft Soils," published in American Society of Civil Engineers (ASCE) Geotechnical Special Publication No. 226, quantitatively include this contribution as a second resisting and stabilizing force.

E. Exit Gradient Instability Potential – Exhibit 7

Exhibit 7 of the Technical Analysis discusses in detail the concerns related to exit gradients induced by upward seepage flow as it emerges at the bottom of an excavation. These gradients, if sufficiently high, could potentially destabilize an excavation bottom. In a granular soil subsurface environment, the nature of this destabilization process would be liquefaction or piping of these soils. In cohesive soils such as the MCF in which the finished excavation would be based for a dry dredging remedial construction, the nature of this destabilizing process is much less severe due to the cohesive shear strength of these soils. In these soils, excessive exit gradients generally manifest themselves by some uplifting/undulating and/or cracking of the excavation bottom surface that is generally considered to be a nuisance rather than a potentially dangerous occurrence as is the case in granular soil excavation bottoms. In addition, soil pore water may emerge as minor seepage flow from developed cracks and fractures in the excavation bottom cohesive soils. This flow can generally be controlled by pumping of the emerging seepage at the excavation bottom using bottom suction pumps.

Two quantitative expressions for assessing the piping instability potential of excavation bottom soils using exit gradient analysis for granular soils were used to simplistically and conservatively evaluate this instability potential for the cohesive soils of the MCF.

F. Rotational Instability Potential – Exhibit 8

As discussed above, basal heave instability potential in clay soils which surround a cantilevered sheet piling retaining structure is typically evaluated using either the quantitative analysis procedure presented by Terzaghi for wide/shallow excavations or by Bjerrum and Edie for narrow/deep excavations. As a cross-check, Luo[6] recommends that basal heave instability potential also be evaluated using a rotational instability model based on an assumed circular arc failure surface geometry within the clayey soils. Luo refers to this analysis as the "slip circle method." The analysis procedure itself was developed by Wu[7] and involves a simplistic hand calculation approach with limited input parameters. Because of the obvious simplicity of the Wu procedure, WESTON instead estimated the potential rotational instability using a much more technically sophisticated Method of Slices based slope stability computer software package developed by Geoslope Inc. (i.e., SLOPE/W). This program is structured to search for the critical failure surface by evaluating a very large number of trial failure surfaces within defined search boundaries. The detailed stratigraphy of the developed subsurface cross sections for both critical test borings AQ-SB-02 and AQ-SB-04 was modelled in the SLOPE/W input files to complete these analyses.

WESTON additionally included in the rotational instability potential analysis, an upward acting uniform surcharge representing the potential destabilizing effects of the underlying artesian pore water pressure in the CFF. This surcharge pressure was attenuated to the elevation of the base of the actual failure mass as described in Appendix B and applied in addition to the surcharge loadings which develop from the weight of the free water and lake bottom sediments on the outside of the sheet piling. As a conservative assumption, the additional "artesian pore water pressure surcharge loading " was only applied to the retained side of the failure mass in the rotational stability model as a downward acting[8] destabilizing force on the failure mass. As suggested by USACE in its review of the analysis procedure, this surcharge could reasonably be applied on both the retained and excavated sides of the cantilevered sheet piling retaining wall, which would effectively result in a counterbalancing effect and negate its influence in its entirety.

[6] Luo, Z; Atamturktur, S. et al; "Simplified Approach for Reliability-Based Design Against Basal Heave Failure in Braced Excavations Considering Spatial Effect;" ASCE, Journal of Geotechnical and Geoenvironmental Engr.; Vol. 138, No. 4, April 2012.

[7] Wu, S.H., et al; "Reliability-Based Design for Basal Heave in a Excavation Considering Spatial Variability;" Proc. Geo Florida 2010: Advances in Analysis, Modeling and Design; Geotechnical Special Publication; 2010, pgs. 1914-1923.

[8] Due to the calculation methods and modeling constraints of the Slope/W computer program, this downward acting destabilizing force had to be applied on the retained side of the cantilevered sheet piling wall to equivalently represent the upward acting destabilizing force resulting from the CFF artesian pressures acting on the failure mass on the excavated side of the structure.

IX. Factor of Safety

To mitigate potential instability, an engineered design will be developed such that the predicted forces which act to stabilize a structure will outweigh those acting to destabilize the structure. This ratio of stabilizing to destabilizing forces is termed the Factor of Safety (FS) and will be greater than 1.0 for a safely designed structure such as the cantilevered sheet piling retaining wall required for this project, or for safely evaluated instability scenarios such as excavation bottom basal heave or bottom uplift. A calculated FS value that is less than 1.0 is indicative of an unsafe design or an unsafe instability potential, and would most probably be indicative of eventual failure should a design be finalized and constructed under those conditions. For additional conservatism, the minimum required FS values for given potential instability scenarios are established at values that are greater than 1.0 by industry standards, thereby insuring that the measured value of the stabilizing forces is significantly greater than that of the destabilizing forces which in turn helps to insure the safety of the proposed construction. For example, the industry recommended minimum required FS values for the excavation basal heave and bottom uplift instability potential analyses are 1.50 and 1.25, respectively. It is noted that Weston's proposed use of these FS values for the excavation basal heave and bottom uplift instability potential analyses for the proposed dry excavation at the Ashland Site have been reviewed and approved by the USACE. Minimum required FS values for the remaining three potential instability scenarios are also established by industry standards and are presented in the following section.

X. Results of WESTON Technical Analyses

WESTON completed a total of five relevant and essential geotechnical engineering Technical Analyses as discussed above to determine the feasibility of a dry dredge remedy at the Ashland Lakefront Superfund Site. These analyses were completed using the subsurface stratigraphy at two plan locations within the proposed dry dredging footprint indicated in the Anchor Evaluation as being "weak links"; that is, at the locations of the offshore test borings AQ-SB-02 and AQ-SB-04 where the lowest FS was calculated against bottom uplift and basal heave instability potential, respectively, among the investigations completed within the dry dredge excavation footprint. As a crucial initial step in standard engineering practice it is prudent to analyze the worst case subsurface soil profile assuming it may exist anywhere on site. This ensures that if the same analysis was performed using a more competent soil profile, a higher FS value will most likely be calculated. The results of the completed Technical Analyses are discussed in detail in Technical Exhibit 9.

Based on the results of the five relevant and essential geotechnical engineering technical analyses for both critical offshore test boring locations, WESTON has concluded that acceptable

FS values that were numerically higher, sometimes significantly, than the minimum required values based on standard geotechnical engineering practice were calculated for all five instability scenarios as shown in the following tabulation. WESTON has therefore concluded that the offshore subsurface environment at the Ashland Lakefront Superfund Site is geologically suitable and favorable for a future dry dredging remedial action project which can be safely completed in the field, and which will be protective of both human health and the environment.

	Factor of Safety		
Potential Instability Scenario	Industry Recommended Minimum FS Value[1]	WESTON Calculated FS (AQ-SB-02)	WESTON Calculated FS (AQ-SB-04)
Structural Stability[2]	1.5	3.1	1.8
Basal Heave	1.5	4.7	1.8
Bottom Uplift	1.25	1.4	1.5
Exit Gradient	2	3.7	5.1
Rotational Instability	1.3	3.7	1.6

1 Industry recommendations for FS values are discussed in the technical analyses and discussions.

2. Structural stability assumes AV 26-700 sized sheets manufactured from 50 ksi strength steel. FS against structural instability potential calculated as yield stress of steel divided by maximum developed bending stress in the section as determined by prosheet.

It is important to note that the tabulated FS values shown above for the subsurface profiles developed from the two critical offshore test boring locations were calculated based on the assumption that the entire, approximately 7.5 acre, dry excavation footprint is sheeted around its perimeter, dewatered, dry excavated and backfilled as one large cell. However, it is common practice for projects of this nature to be completed by instead creating smaller sheeted cells within the larger sheeted footprint using internal lines of sheet piling which structurally connect to the perimeter sheeting. This remediation concept permits a much more easily managed, efficient and cost effective staged construction approach to be used instead, in which one cell at a time can be remediated. In this regard, WESTON's completed technical analyses have also demonstrated that, as the area of a sheeted dry excavation cell is reduced, the FS against the bottom uplift instability scenario increases, while the FS values for the remaining four instability scenarios remain the same. Therefore, Weston has also concluded that completing the dry excavation remedial action at the project Site using a to- be-determined number of smaller sheeted cells rather than one large cell is also technically feasible and can be safely completed in the field using standard engineering and construction practices, and will be protective of both human health and the environment.

A future dry excavation remedial action will not only be required to remove contaminated sediments and miscellaneous fill materials beneath the free water of Chequamegon Bay, but also at shoreline locations along the face of an anchored bulkhead wall installed previously. Although free water is not present at these locations, the instability potential scenarios discussed previously in this report, and at length in the Technical Analysis, are also relevant to excavations conducted at these locations. The Anchor Evaluation additionally presented FS values for bottom uplift instability for 14 borings completed by the Foth/Envirocon Joint Venture in 2012. Of these 14 borings, SB-185 is indicated in this report as having the lowest FS against excavation bottom uplift, a value lower than reported for the offshore boring AQ-SB-02. Since SB-185 exists beyond the extent of the excavation footprint and may not be intimately representative of the dry dredge footprint subsurface soils, the determination of an appropriate MCF thickness to be used in evaluating the essential stability analysis of bottom uplift is left to the discretion of the final design engineer. However, as a means of comparison to the Anchor Evaluation, WESTON also completed an independent technical evaluation of the subsurface environment relevant to test boring SB-185. This technical evaluation is presented for illustrative purposes only and a detailed discussion of these analyses is presented in Technical Exhibit 9. The results of these analyses indicate that a dry dredge excavation remedial construction may be safely completed in the vicinity of SB-185 using a reduced excavation footprint consistent with standard construction practices as discussed above. For example, by subdividing the larger excavation footprint of over 320,000 ft^2 into a more manageable cell with an approximate footprint of 20,000 ft^2, a FS against excavation bottom uplift is calculated at an acceptable value of 1.26. This evaluation of SB-185 very conservatively assumes that a comparatively thin MCF thickness of 25.5 feet exists over the entire analyzed excavation footprint. As discussed in the Technical Analysis, if a greater, more representative MCF thickness value were to be used for this analysis, the FS would increase from the values presented.

It must be noted that unanticipated subsurface conditions may be encountered during the dry excavation field work which would have the effect of reducing one or more of the calculated FS values for the five instability scenarios to unacceptable values. An example of this occurrence would be the discovery during the dry excavation construction work that the thickness of contaminated bay bottom materials targeted for removal is considerably thicker than anticipated within a localized area of the dry excavation footprint. This would obviously correlate to a much thinner remaining MCF within this localized area, which in turn would have the direct effect of reducing the FS value against the development of an excavation bottom uplift instability. In this instance, other engineering controls are available to the Engineer which could be quickly implemented to counteract this destabilizing effect. For example, groundwater dewatering wells screened within the CFF could be installed within this localized area of the dry excavation footprint. Subsequent pumping of these wells would lower the artesian head and corresponding uplift pressures at the MCF/CFF interface, thereby increasing the FS value against the development of an excavation bottom uplift instability. This process may be completed actively through pumping or passively relying on the artesian pore water pressures to drive pore water vertically upward through the well points to the dry excavation bottom ground surface. The removal of water within this localized footprint would continue until the reduced artesian pore pressure head within the CFF is sufficiently lowered such that the calculation of an acceptable FS value (i.e., a value greater than 1.25) results from the reanalysis of the instability potential including the reduced artesian pore pressure and MCF thickness. At this time the deeper excavation could then be safely completed within this localized area. The collected CFF waters removed from the excavation may be re-injected into the CFF at up-gradient on-land locations to minimize the dewatering effects on the local groundwater hydrology.

It is common, especially when working in wet environments, to experience localized non-critical manifestations of what may be considered quasi bottom uplift or exit gradient instabilities where the excavation bottom may deflect slightly upward on the order of several inches, or pore water within the soils at the top of the exposed MCF stratum may actively seep into the excavation bottom. These developments are non-serious and commonplace in excavation work, and are typically and easily remedied by staged excavation and bottom grading activities, and/or active dewatering of the excavation bottom using readily available bottom suction pumps.

XI. Construction Approach

As discussed previously, the tabulated FS values for the five relevant and essential geotechnical instability scenarios of the offshore subsurface environment considered a full excavation footprint approach in which sheet piling would initially be installed around the 7.5 acre perimeter of the dry excavation footprint, followed by dewatering, dry excavation of contaminated materials, and backfill of the excavation zone. As also discussed above, it is more practical and efficient to instead create sheeted subdivided cells within the 7.5 acre footprint to complete the dry excavation construction work. Weston's Technical Analysis have also demonstrated that as the sheeted excavation footprint is reduced, the FS against bottom uplift increases, while all other FS values remain the same.

The major components of a cell by cell remedial construction approach are briefly and generally defined as follows:

1. Install the 200 foot offshore sheet pile retaining structure using a driving rig positioned on a barge.
2. Install the first internal sheet piling cell separation wall from the shoreline to a structural connection with the 200 foot offshore wall. The width of the cell should be selected such that a long reach excavator positioned on a constructed geosynthetic reinforced unpaved road located along the half width alignment of the cell footprint can reach and excavate all contaminated materials located inside the perimeter of the cell.
3. Install two more cell separation sheet piling walls adjacent to the initial cell.
4. Dewater the first cell.
5. Perform shoreline grading and install a temporary road within the cell footprint with excess site soils and geosynthetic reinforcement to allow the excavator to enter the dewatered cell.
6. Begin dewatering second cell.
7. Excavate contaminated offshore sediments in the first cell and load onto haul trucks using the same temporary unpaved road as the excavator. Continue excavating sediments followed by removal of the unpaved road and the contaminated materials which directly underlie the footprint of the road working from the furthest offshore location of the road back towards the shoreline until all confirmation sampling indicates complete removal of the contaminated materials.
8. Perform shoreline grading and temporary unpaved road installation in second cell subdivision when dewatered.
9. Move excavation equipment and personnel to second cell and begin excavation activities while using different equipment to place and compact restorative layer, proceeding from shoreline to excavation extents in the first cell.
10. Reflood and remove non-shared sheet piling from the first cell. Install removed sheet piling adjacent to the third cell in the same manner as the first three cells.
11. Continue steps 6 through 10 in a progressive manner within additional subdivided cells along the Site shoreline until dry dredge footprint has been remediated.
12. Pull remaining sheeting as final cells are completed.

It is finally noted that limiting the active excavation footprint to a manageable size allows for an assembly line procedural approach that maximizes crew efficiencies and decreases equipment and sheeting rental quantities, which will in turn decrease overall costs of the remedial action. This approach additionally reduces free water dewatering downtime and the volume of seepage inflow into the open excavation that would likely need to be treated.

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